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Vol. X JANUARY, 1923 No. 1

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PROCEEDINGS

PAPERS IN THIS NUMBER.

"Methods of Design and Construction and Results of Operation of Submerged Pipe Lines for Outfall Sewers." Report of Committee.

"Some Features of Sewer Design." By John W. Raymond, Jr.

"Foundations for the new First National Bank of Boston." By Howard K. Alden.

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Contributors are hereby notified that proof will not be submitted to them for examination unless requested before the 10th of the month preceding the month of publication.

MINUTES OF MEETING.

Boston, December 20, 1922. — A regular meeting of the Boston Society of Civil Engineers was held this evening in Lorimer Hall, Tremont Temple, and was called to order by the President, Dugald C. Jackson, at 7.45 p.m. There were about 150 members and guests present.

The minutes of the previous meeting were approved as printed in the November Journal.

The Secretary announced the names of those elected to membership by the Board of Government at meetings held on November 23, 1922 and December 20, 1922.

November 23, 1922,

Members — Messrs. Edward Sherman Chase, Howard Pike Fessenden, Thomas P. Glynn and Edward L. Moreland.

Juniors — Messrs. Benjamin Warder Thoron and Earl Holden Winslow.

December 20, 1922,

Members — Messrs. William J. Alcott, Jr., Everett A. Armington, Ralph E. Baker, Howard L. Brown, John J. Burke, John H. Burroughs, Daniel J. Cadigan, Charles C. Carroll, William N. Carroll, Hiram R. Chubbuck, Samson K. Cohen, William F. Donnelly, Thomas J. Fitzgerald, Lawrence R. Flynn, Edwin T. Hoisington, Arthur W. Hunt, Thomas Kelly, Thomas B. Kenney, Arthur J. Lennon, William A. Liddell, Frank P. McCarthy, Edward L. Moulton, Frederick F. Murphy, James S. Murray, John H. L. Noyes, Michael D. O'Farrell, Alonzo B. Reed, Elwyn L. Russell, Richard A. Scannell, George H. Sherman, Roland G. Stafford, Frank B. Stevens, Jr., Thomas F. Sullivan, William J. Sullivan, Lawrence E. Weeks, Kilborn Whitman, Jr.

Juniors — James H. Bartlett, Jr., Emery W. Garney, Saul E. Moffie.

The President announced the death of Robert C. P. Coggeshall on October 20, 1922, and of Lawson Bennett Bidwell on November 19, 1922. Mr. Bidwell was President of the Society in 1901-02.

It was voted to authorize the President to appoint committees to prepare memoirs of these members.

The subject of the evening was "The Manufacture and Distribution of Gas" and Herbert N. Cheney, Vice-President of the Boston Consolidated Gas Co., gave a brief history of the development of the Gas Industry up to the present time.

He next described in detail the operation of the model water gas plant which had been set up on the stage. This plant, built to the scale of one inch to the foot, and complete in every detail, was then operated by D. S. Reynolds, Engineer of Construction of the Boston Consolidated Gas Co. Several runs were made showing the complete cycles involved in the manufacture of water gas, the various stages being explained by means of an electric sign which described each step as it took place.

The second portion of the program was an illustrated talk on "Gas Holder Design and Construction," by E. F. Miller, Consulting Engineer (formerly Chief Engineer, Camden Iron Works). Mr. Miller traced the development of Gas Holders from the early types to the most modern ones under construction at the present time.

After further questions concerning the water gas plant operation and a brief discussion of Mr. Miller's paper, the meeting adjourned at 9.50 P.M.

J. B. BABCOCK, *Secretary.*

APPLICATIONS FOR MEMBERSHIP.

[January 15, 1923.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

FAY, HERMAN, Hartford, Conn. (Age 27, b. Danvers, Mass.) He has taken two courses with Wilson Engineering Corporation, concrete engineering and design, and structural engineering. Has had three years as foreman and superintendent on building construction; two years as draftsman and designer; one and one-half years in the army in charge of construction; and two years with Edward A. Tucker Co., as draftsman, designer, inspector and is at present manager of the branch office. Refers to W. L. Cronin, W. S. Fox, E. A. Tucker and J. F. Wilber.

Foyer, Charles Pearson, Boston 25, Mass. (Age 34, b. Liverpool, Eng.) Educated in the Boston English High School and Northeastern University Y. M. C. A., evening school. In August, 1906 he entered the employ of the N. Y., N. H. & H. R. R. as rodman on railroad maintenance work; in December, same year, he entered the construction department engineer's office as instrumentman, and has been with this railroad ever since, with the exception of twelve months. Is now assistant engineer under A. S. Tuttle. Refers to L. G. Brackett, H. L. Ripley, A. S. Tuttle and B. T. Wheeler.

French, Grant Keith, Lowell, Mass. (Age 25, b. Lowell, Mass.) Graduate of Lowell High School and Mass. Inst. Tech. During the summers of 1919, 20 and 21, he was assistant instructor at Tech. summer school, and during the academic years was an assistant instructor in civil engineering at Tech. During this time he worked on various surveying jobs both in the office and field. Since June, 1915 he has been assistant on the engineering corps of the Chicago Terminal Division of the Penn. R. R., on surveying, estimating, drafting, checking computations, designing track layouts, etc. Refers to C. B. Breed, G. L. Hosmer, J. W. Howard, C. M. Spofford and C. H. Sutherland.

MacAffee, John, Cambridge, Mass. (Age 29, b. Woodstock, N. B.) Educated in the public schools and graduate of Tufts College. 1910-15 he was with the Canadian Pacific R. R. on maintenance and construction work; 1916 with French & Hubbard, hydro-electric survey; 1917 with the British government; 1918 instrumentman and chief of party with Fay, Spofford & Thorndike, on Boston Quartermaster's Terminal construction; 1919-1920 assistant chief engineer on town layouts, mill, streets, dam, pumping station, etc., in Cuba 1921 with the J. G. White Management Corps, Central America; 1922 with the N. Y., N. H. & H. R. R., construction department. Refers to H. F. Heald, H. F. MacWilliams, J. F. Wilber.

Nickerson, Frederick Prince, Needham, Mass. (Age 37, b. Centerville, Mass.) Had two years in the High School, also studied in Y. M. C. A. evening school and Correspondence School. He has had experience as chainman on proposed electric railway survey from Sandwich to Hyannis; 1905-06 chainman and rodman with Aspinwall & Lincoln; 1906-09 employed on the Penn. Tunnel & Terminal Co.; 1909-10 transitman on preliminary survey of electric railway from Detroit to Lansing; 1910-12 with N. Y., N. H. & H. R. R. as leveller and transitman; 1912-13 instrumentman on B. & M. R. R.; 1913-14 engineer with T. Stuart & Son Co.; 1914-16 transitman with the B. & M. R. R. as transitman, resident engineer and assistant engineer on construction work. Refers to L. G. Brackett, F. C. Shepherd, A. S. Tuttle and B. T. Wheeler.

Proctor, Thomas White, Chestnut Hill, Mass. (Age 26, b. Boston, Mass.) Graduate of Dartmouth in 1918, and Mass. Inst. Tech. in 1921, C. E. Dept. August 1921 to May 1922 was chainman on the Main Central Railroad; May 1922 to December was transitman for H. K. Barrows, on dam construction at Salem, N. H., and pipe line survey at Gardiner, Mass. Refers to J. B. Babcock, H. K. Barrows, C. B. Breed, C. A. Moore, D. Porter and C. M. Spofford.

Reynolds, Kenneth Cass, Somerville, Mass. (Age 25, b. Somerville, Mass.) Graduate of Tufts College 1919 in civil engineering. September 1919 — October 1920, with Fay, Spofford & Thorndike as transitman, draftsman and designer; October 1920 to October 1921 was assistant in the C. E. department at Mass. Inst. Tech.; October 1921 to date instructor in civil engineering at Mass. Inst. Tech.; Refers to H. K. Barrows, E. Harsch, D. Porter, G. A. Sampson, H. F. Sawtelle and W. D. Trask.

Riordan, Timothy P., Roxbury, Mass. (Age 27, b. Boston, Mass.) Graduate of Sherwin Grammar School 1911, and spent the next two years in Killarney Seminary, Killarney, Ireland. 1914-1921 employed by the city of Boston as rodman (with the exception of eighteen months service in U. S. Marine Corps); 1921 to date transitman in the paving department, city of Boston. Refers to B. F. Bates, J. E. Carty, T. J. Haggerty, P. J. Malley, C. G. Norris and J. H. Sullivan.

LIST OF MEMBERS.

ADDITIONS.

Armington, Everett A.....................44 Dean St., Everett, Mass.

Baker, Ralph E.................104 Frederick Ave., Medford, Mass.
Brown, Howard L.................... 5 Tainter St., Medford 55, Mass.
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Russell, Elwyn L.................404 City Hall Annex, Boston, Mass.
Stafford, Roland G................321 North Station, Boston, Mass.
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Sullivan, William J...........15 Manthorne Rd., West Roxbury, Mass.
Weeks, Lawrence E................477 Highland Ave., Malden, Mass.
Whitman, Kilborn, Jr..............109 Walnut Ave., Boston 19, Mass.

Death.

Reed, Carl B.......................................December 29, 1922.

LIBRARY NOTES.

Book Review.

"The Young Man and Civil Engineering," by Professor George Fillmore Swain, LL.D. The Macmillan Company, New York.

Reviewed by George Edmond Russell.*

This admirable book is one of a series of twelve books written, or to be written, by men of large knowledge and experience for the purpose of aiding and guiding young men who are searching themselves and the vocations in order to choose a field of life endeavor which shall be best coördinated with the tastes, qualities and personal characteristics which they find to be their own.

The series, edited by E. Hershey Sneath, Ph.D., LL.D., Yale University, and written by eminent representatives of the different vocations, is as follows:

1. The Young Man and the Law.
   Hon. Simeon E. Baldwin, LL.D.
2. The Young Man and the Ministry.
   Rev. Charles R. Brown, D.D., LL.D.
3. The Young Man and Teaching.
   Professor Henry Parks Wright, Ph.D., LL.D.
4. The Young Man and Medicine.
   Lewellys F. Barker, M.D., LL.D.
5. The Young Man and Journalism.
   Chester Sanders Lord, M.A., LL.D.
6. The Young Man and Banking
   Hon. Frank A. Vanderlip, M.A., LL.D.
7. The Young Man and Business.
8. The Young Man and Mechanical Engineering.
   Lester P. Breckenridge, M.A., Eng.D.
9. The Young Man and Electrical Engineering.
   Charles F. Scott, Sc.D., Eng.D.
10. The Young Man and Civil Engineering.
    George F. Swain, LL.D.

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* Professor of Hydraulics, Mass. Inst. of Tech. Cambridge, Mass.

11. The Young Man and Farming.
    L. H. Bailey, M.S., LL D.
12. The Young Man and Government Service.
    Hon. William Howard Taft, D.C.L., LL.D.

The reviewer has not had the opportunity to read other books of the series, but if the "Young Man and Civil Engineering" may be taken as indicative of the quality of the others, he unhesitatingly recommends the series to every young man of sober thought and earnest purpose.

The book is of two hundred pages embodying the following contents.

Chapter

I. Historical Introduction.
II. Branches of Civil Engineering.
III. Qualifications Necessary or Desirable for the Civil Engineer.
IV. The Education of the Civil Engineer.
V. Characteristics of Civil Engineering as a Profession.
VI. The Outlook for the Civil Engineer.
VII. Concluding Suggestions.

The reader will be impressed by the forceful treatment of the various subjects and the clarity and conciseness with which the author handles each detail. He will feel at once that here is a book by a man who knows his subject and knows how to put it before his reader in a manner that is both interesting and convincing.

The brief space allowed here for this review prevents the taking up of each chapter in detail. The first two chapters probably contain nothing which will be really new to the well read engineer, but the style in which the author introduces and develops his subject makes these chapters not only of interest to the young man for whom they were written but also to the professional engineer no matter how far advanced.

Chapter III discusses the necessary professional and personal qualifications for the engineer —

"Professional qualifications are those qualities or acquirements which affect mainly his *knowledge*, and which, therefore, supply the basis for his intelligent dealing with the problem to be solved. Personal qualifications are those qualities which fit him to apply his knowledge properly, economically, and effectively, and so to accomplish the result which his professional knowledge and skill indicate to be desirable or necessary. The two kinds of qualifications do not always go together, yet their union is necessary for the highest success. An engineer may have great professional ability, yet on account of the lack of some personal quality, such as tact or promptness, may be a miserable failure in certain kinds of work."

Under desirable personal qualifications the author makes the following list:

(1) Judgment,
(2) Balance,
(3) A Trained Mind,
(4) Experience,
(5) Initiative,
(6) Good Health,
(7) Knowledge.

The position of knowledge at the end of the list momentarily raises a question in the reader's mind but it is soon convincingly answered.

In the "Education of the Engineer," Chapter IV, the author is decidedly at his best. He shows the true value of a college education and its relation to the other factors which make for success. He emphasizes balance and breadth in the selection of studies to be followed, points out the ills of the elective system, and urges the student to avoid the narrowing influences of overspecialization.

"Select courses that will call forth your most serious effort, not those that are easy for you. Do not *aim* for culture, whatever you may understand that to mean, but for training. . . . Do not go too deeply with any one subject, but study the fundamentals thoroughly. You cannot become a specialist or an expert in four years. If you try to become an expert in college, you will more likely turn out a sham or narrow bigot. If you *aim* for culture, you will be likely to turn out a conceited ass; if you *aim* for wisdom, you will be likely to turn out a pedant. If you work hard, take advantage of your opportunities, follow good advice, culture and wisdom will come of themselves in due time, if they are meant for you."

The last chapter, "Concluding Suggestions" should be reprinted and put into the hands of every student in our technical schools. It is filled with good sound advice, honestly given out of the depth of a broad engineering and teaching experience. Written for the young beginner, it is a true guidepost on the road to success; read by the "hardened sinner" in professional old age it makes him almost unknowingly wish he might find the water for renewing youth and begin again his upward struggles toward success.

The book should be studied in order to be appreciated. It should be in every civil engineering school and a wise employer would see that it came into the hands of all young men in his office.

Recent Additions to the Library.

U. S. Government Reports.

Financial Statement of Cities of over 30 000 population in U. S. 1921.

National Research Council Bulletin.

Statistics of Railways in United States 1920.

United States Coast and Geodetic Survey, Report 1921.

State Reports.

New York. Report New York State Bridge and Tunnel Commission. 1922.

Massachusetts. Annual Report Department of Public Health. 1921.

Rhode Island. Public Utilities Commission Report. 1921.

Municipal Reports.

Boston, Mass. Annual Report City Planning Board. 1921.

Woonsocket, R. I. Annual Report Board of Water Comissioners. 1921.

Miscellaneous.

Development of Scientific Rates for Electricity Supply. Edison Illuminating Co., Detroit.

Library Committee.

Vol. X JANUARY, 1923 No. 1

BOSTON SOCIETY OF CIVIL ENGINEERS
FOUNDED 1848

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications

METHODS OF DESIGN AND CONSTRUCTION AND RESULTS OF OPERATION OF SUBMERGED PIPE LINES FOR OUTFALL SEWERS.

REPORT OF COMMITTEE.

(Presented at a meeting of the Sanitary Section on March 1, 1922.)

YOUR Committee has received much valuable information on this subject from various sources. The data include detailed descriptions of about forty outfalls, together with several plans, specifications and photographs which are on file at the library of the Boston Society of Civil Engineers. It is the purpose of the Committee to present only a summary of the salient facts which the data disclose.

LOCATION.

Submerged outfall sewers are used to dispose of sewage in those cases where large bodies of water offer the opportunity for disposal by dilution and oxidation. They are therefore found on the seacoast, Great Lakes and large rivers. The proper method for pre-treatment of the sewage before disposal by a submerged outfall, depends on the dilution afforded, the condition of tides and currents, the character of the sewage, the proximity of bathing beaches and residences, and other local considerations.

The treatment varies from none whatever, to Imhoff tanks followed by filtration. Ordinarily, when the outlet can be located at a depth of 30 ft. or more below low water, and the direction of current is favorable, it is sufficient to remove or comminute the solids by subsidence, screening or otherwise. On the New Jersey coast, the clarification or so-called " septic " tank has been used with good results, and seems preferable to the Imhoff tank because of the greatly varying period of detention between winter and summer use.

A check valve is required on the outlet of the tanks affected by tides, to prevent the entrance of salt water.

The governing condition of the design is to extend the outfall to a point where the water is deep enough to effect a satisfactory degree of dispersion before the sewage reaches the surface, and to be beyond the destructive action of the waves and the danger from shifting sand. A depth of not less than 30 ft. is preferable.

MATERIAL.

Pipes of cast iron, wrought iron, the latter galvanized or plain, and riveted steel, are almost universally used, although some large reinforced concrete outfalls have been recently constructed. A few wooden outfalls are reported, but they are short, and not typical of the kind that we are discussing, i.e., long submerged outfall that effects final disposal of sewage in deep water.

The life of ordinary steel pipe in contact with salt water is probably too short to warrant its use in permanent outfalls. The life of plain wrought iron pipe is from 15 to 20 years, which may be somewhat prolonged by galvanizing, although there is some doubt as to whether the extra expense is warranted. Cast iron and reinforced concrete are the most durable.

SIZES AND LENGTHS.

Sizes of pipe ranging from 6 in. to 84 in. in diameter are reported, and lengths from 36 ft. to 10 000 ft. The size depends upon the quantity of sewage to be conveyed, and the velocity resulting either from gravity flow or pumping. A self-cleaning velocity should be maintained or some provision for flushing be installed. The length is governed by the profile of the bottom, the depth decided upon for the outlet, the proximity of beaches, and the volume of sewage as affecting dilution.

OUTLETS.

The location of the outlet end is governed by four considerations —

(*a*) Stability of the bottom,
(*b*) Depth of water,
(*c*) Distance from beach,
(*d*) Prevailing currents.

(*a*) *Stability of the Bottom.* It is imperative that the outlet end of the pipe should be carried far enough out so as to reach a point where the bottom is stable. This will be found where the water is comparatively deep, beyond the zone of disturbance occurring near the beach where the wave action is most violent. If the outlet is within this zone, it is liable to be destroyed or buried up in shifting sand.

(*b*) *Depth of Water.* It is desirable that the discharge of sewage should take place in the deepest water practicable so as to cause quick dispersion and minimize the apparent nuisance.

(*c*) *Distance from Beach.* The outlet should be carried out as far as necessary to prevent visible pollution reaching the beach, and if the latter is used for bathing, the outlet should be so far removed that the bacterial count of the beach water is not materially increased.

(*d*) *Prevailing Currents.* The prevailing current should be ascertained by submerged and surface float experiments, and a point selected for the discharge where the current is off shore all or most of the time.

Any one or more of these considerations may govern the location of the outlet, and therefore determine the length of the outfall. As an aid in studying these questions, soundings should be made over the areas considered as possible outfall locations, and a profile taken on the line of the proposed outfalls.

The outlet end of the pipe is usually secured to an anchor of masonry or piles and crib work or some such suitable combination, and turned up either vertically or at an angle of 45° from 4 to 6 ft. above the bottom. Sometimes the discharge is horizontal through one or more outlets. A cast iron anchor block secured to the masonry is desirable to hold this bend. If a concrete anchor block is used, the concrete should be thoroughly mixed in the proportions of one quick-setting, natural, hydraulic cement, to six fine and coarse aggregate, combined to give a dense mixture, and carefully placed by means of a water-tight funnel extending down to the fresh concrete.

In the great majority of cases, the pipe is carried full size to the end, and the discharge all takes place at this one opening, but in some cases, particularly when the water is shallower than is desirable, there are top openings provided near the end, and the bore of the pipe is correspondingly reduced for the obvious purpose of getting wider dispersion, and to maintain a self-cleaning velocity. In a few cases, the outfall pipe is divided at the outlet end into a number of smaller pipes for the same reason.

Preparation of the Bottom.

Only in the cases of the larger pipes is there usually any preparation of the bottom to receive the pipe. In the case of the large, 84-in., C.I. pipe of the Metropolitan Sewerage System at Nut Island, a trench was dredged, and pile bents driven to support the pipes. The pipes were laid in this trench, and the jointing done by divers. In the case of the smaller pipes, however no digging is required except through the crest of the beach and down to low water, and in special cases where there is danger of injury from boats' anchors or otherwise when the pipe is laid in a trench or protected by riprap.

When a submerged outfall extends beyond a pier head, which is usually advisable, to a distance of at least 50 ft., it should be laid without obstructing either the actual or legal channel or waterway, and so as to avoid injury by dredging operations or anchoring.

Joints.

If cast iron pipe is used, the joints should be made with lead, and every third joint should be ball and socket, both for the smaller sizes to prevent breakage in launching as described below, and for the larger sizes which are laid in place by sections in order to conform to the contour of the bottom.

If wrought iron or steel pipe is used, the joints are usually screw joints, sometimes covered with lead-filled sleeves several feet long. This is good practice on the joints between high and low water where there is most liability to injury from the pounding of the breakers. It is also good practice to introduce a telescoping joint near low-water mark, which may be provided in connection with one of the sleeve-covered joints.

Some difficulty has been encountered where wrought iron and steel pipes are laid on pile bents, by the pipe rubbing against the bents until holes occur, for which reason this method of securing the pipe is questionable. In any case, where pile bents are necessary, or a timber crib work is used at the outlet, all bolts should be galvanized, and all washers cast iron.

LAUNCHING.

The pipe, including the cast iron outlet block, is usually assembled on shore on the prolongation of the line upon which it is desired to deposit it in the water. Temporary ship's anchors are fixed in the bottom on the same line and at a considerable distance beyond, i.e. to seaward of the point where the outlet end is to be permanently anchored. A large sheave is attached to this temporary anchor, and a cable reeved through it, one end of the cable being attached to the outlet end of the pipe, and the other end to a large winch on shore. The pipe is supported on rollers or trucks, and is pulled out into position by the winch.

With wrought iron pipe it is necessary to provide ballast to prevent excessive buoyancy and injury by waves, which is accomplished by running wrought iron pipe inside and filling the latter with water.

With cast iron pipe, some means of buoyancy has been found advantageous. Two methods have been used to support the pipe in the passage from the beach to the permanent anchor point — one by slinging it from pairs of boats, and the other by supporting it on low trucks with thick wooden wheels, with devices for releasing the trucks when the pipe has arrived in its permanent position. In order to prevent fouling, it has been found useful to buoy the return cable from the sheave attached to the temporary anchors to the winch with oil barrels. The casting holding the raised outlet end of the pipe is deposited upon the permanent anchorage and secured thereto by divers.

COST.

The difficulty of presenting any definite figures as a guide in estimating the cost of submerged outfalls will readily be appreciated. Some of the factors influencing the unit cost of any outfall are the kind of pipe determined upon, length of submerged line, depth of outlet below mean low water, velocity of tide or stream, character of the bottom, atmospheric conditions at time of construction, method available for laying the pipe, and special provisions necessary to conform to sanitary regulations, etc. From our study of existing outfalls, we have, however, deduced certain costs which are given in the following table and may be considered to represent average pre-war conditions. Under extreme circumstances, the costs were in a few instances 100 per cent. in excess of and 50 per cent. less than those below.

Size of Outfall.	Price per Foot Including Outlet.
8 in.	$5.00
12 in.	6.25
16 in.	7.50
24 in.	10.25
36 in.	14.75
48 in.	20.00
60 in.	26.00
72 in.	32.50
84 in.	39.50

OPERATION.

These submerged outfalls are reported, with very few exceptions, to have given satisfaction. Several instances are recorded where it is difficult to locate the outlet from any surface indication; others where the surface is discolored with a scum over but a small area immediately above the outlet; and some, however, where the discoloration extends for several hundred feet, although not objectionable. Such troubles as have been experienced, have arisen from two sources: —

(*1*) Displacement or burying of the outlet due to its not having been carried out far enough to reach a stable bottom in comparatively still water, and beyond the region of shifting sands.

(2) Displacement and breakage of the pipe on the beach, due to the pounding of the waves and cutting away of the sand, leaving the pipe unsupported.

The remedy in the first case is obvious. The best remedy which seems to have been found for the second case is to enclose the exposed joints in long sleeves in some cases 12 ft. in length. These sleeves are made of iron or steel pipe split and flanged longitudinally and are packed with lead. One such protected joint should be made to telescope to provide for longitudinal expansion and contraction. The pipe should be buried above low water, and protected so that it will not be undermined by wave action, and should be laid with gentle changes of grade. The manhole at the shore end of the outfall should be sufficiently removed from the beach to be safe against storms. Careful inspections should be made of outfalls for deposits, leaks, damage by storms and other adverse conditions for satisfactory operation, and the trouble corrected at once.

If attention is paid to the points discussed above, and the proper precautions taken, there would seem to be no reason why an engineer should hesitate to install a submerged outfall sewer when this method of sewage disposal will satisfy the sanitary requirements of the case.

Respectfully submitted,

HIRAM A. MILLER, *Chairman.*
EDGAR S. DORR
GEORGE A. SAMPSON.

Vol. X JANUARY, 1923 No. 1

BOSTON SOCIETY OF CIVIL ENGINEERS
FOUNDED 1848

PAPERS AND DISCUSSIONS

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SOME FEATURES OF SEWER DESIGN.

By John W. Raymond, Jr.,* Member Boston Society of Civil Engineers.

(Presented before the Designers Section, April 12, 1922.)

In sewer design as in other branches of engineering, changes and improvements are continually being made, and what may have been considered good practice a few years ago may possibly in some respects be so no longer. Sewer design is by no means a mechanical process by which the engineer has only to use accepted formulas and grind out the results. Many articles upon this subject have been published and committees of engineering societies have collected and studied data on run-off and rainfall, but definite conclusions, reliable formulas, or methods which would make of sewer design a routine process have apparently not been derived. The reports of run-off committees show that there is a lack of reliable data and a need that engineers and public officials coöperate to improve and enlarge upon such as are available A collection of some of the best data on run-off in sewers is contained in the Report of the Run-off Committee of the Boston Society of Civil Engineers.† In general each engineer has his own method of sewer design, based onhis own experience and judgment. The methods described below, therefore, represent the process as used by a comparatively limited number.

Before proceeding to the details of design it may be helpful to give definitions of some of the more common terms used in practice. These were recommended by the Committee on Sewerage and Sewage Disposal of the American Public Health Association, in order to avoid the confusion caused by calling different things by the same name, or the same thing by different names.

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* With Metcalf & Eddy, Consulting Engineers, 14 Beacon Street, Boston, Mass.
† Journal Boston Society of Civil Engineers, Vol. 1, 1914, p. 291.

1. " *Sewage* is a combination of the liquid wastes conducted away from residences, business buildings and institutions, together with those from industrial establishments; and with such ground, surface and storm water as may be present.

2. " *Domestic sewage* is that from residences, business buildings or institutions.

3. " *Industrial wastes* are the liquid wastes resulting from the processes employed in industrial establishments.

4. " *Surface water* is that portion of the precipitation which runs off over the surface of the ground.

5. " *Storm water* is that portion of the precipitation which runs off over the surface during a storm and for such a short period following a storm as the flow exceeds the normal or ordinary run-off.

6. " *A separate sewer* is a sewer intended to receive domestic sewage and industrial wastes without admixture of surface or storm water.

7. " *A combined sewer* is a sewer intended to receive domestic sewage, industrial wastes and surface and storm water.

8. " *A drain* is a conduit, for carrying off storm water, surface water, subsoil or ground water, (domestic sewage and industrial wastes being excluded).

8*a*. " *A storm drain* is a conduit for carrying off surface water and storm water.

A lateral sewer or drain is one which does not receive sewage, surface or storm water from another sewer or drain.

A sub-main or *branch* sewer or drain is one which receives the flow from two or more laterals.

A main or *trunk* sewer or drain is one which receives the flow from two or more sub-mains.

9. " *An intercepting sewer* is a sewer generally laid transversely to the general sewer system to intercept all the sewage collected by the sewers of a separate system or the dry weather flow of sewage and such additional surface and storm water as may be determined from a combined system.

10. "*A relief sewer* (or drain) is a sewer designed to carry a portion of the flow from a district already provided with sewers (or drains) of insufficient capacity, and thus prevent overtaxing the latter.

11. "*An outfall sewer* is a sewer extending from the lower end of the collecting system to a point of final discharge into a body of water, or to a sewage treatment plant."

Combined versus Separate System. One of the first decisions which the designer is called upon to make is whether the combined or the separate system is best suited to the needs of the district. The main factors controlling this decision are —

1. Conditions at and below the point of discharge,
2. Means of disposal of storm water,
3. Cost of construction,
4. Operation difficulties.

Where the sewage or a small proportion of it can be discharged into a stream or other body of water, without danger to water or ice supplies and without probability of causing other objectionable conditions, the combined system may be chosen. Where such in not the case but streams, which may receive storm water alone, are located near the area to be served, the separate system will probably prove best.

The cost of a combined system is usually less than that of a separate system, because with the former but one conduit is required in each street instead of two with the latter system. Nevertheless, the separate system will sometimes prove the cheaper, particularly for a district containing small brooks or bodies of water, which may receive the flow from drains, provided domestic sewage and industrial wastes are rigidly excluded. Also small communities may require disposal of the domestic sewage and industrial wastes to some distant point yet they may not be able to finance the cost of sewers of sufficient size to care for the storm water, in which case the separate system may be chosen — conduits for sewage alone being installed, the storm water being permitted to run away through the gutters to natural channels. It often happens, however, that due to lack of proper inspection while making building connections, or due to lack of support from higher city officials, the regulations regarding such connections cannot be enforced, and sewage is admitted to drains and storm water to separate sewers, resulting in pollution of the brooks and overtaxing of the separate sewers. This in effect defeats the purpose of the separate system and we have the brooks practically converted into combined sewers. It is apparent that the combined system is the simpler and more satisfactorily administered than the system of separate sewers and storm drains.

Quantity of Sewage. The quantity of sewage depends upon the amount of municipal water consumption, the consumption from independent water supplies, the population, the character and number of industries and the quantity of ground water likely to reach the sewer.

The municipal water consumption is commonly expressed in gallons per capita daily. This may vary from below 50 to higher than 150 gal. per capita per day, depending upon the number of services metered, the type of residences and whether there are large industrial users of water. The tendency during the past few decades has been toward a gradually increasing rate of consumption. The consumption varies widely throughout the day. Whereas the consumption may average 100 gal. per capita per day it may reach a maximum of 150 at noon and in the early morning hours it may drop as low as 75. In sewer design it is the maximum momentary or " peak " rate which must be provided for, excluding such exceedingly excessive rates as may be required for fires or street flushing. Thus if it is found that the average daily consumption is 100 gal. per capita per day, it may be prudent to allow for 200 gal. per capita per day in computing the required capacity of sewer, in order to allow for the tendency for a higher rate of consumption and for the fluctuations during the day.

The density of population which varies widely in different parts of a city may materially affect the quantity of sewage per unit area for which provision must be made. For example, in residential districts of single detached houses, completely built up, the density may be 30 to 50 persons per acre, yet there may be small areas containing schools or tenement blocks or apartment houses where the population may exceed 100 persons per acre. Generally the larger the area, however, the lower the average density of population. Studies of variations in density have shown that in some cities it varies approximately as a straight line on logarithmic paper. Thus it may be found that on ten acres there are 100 persons per acre and on 500 acres the average is 36 persons per acre, and on 1 000 acres the average may be only 30 persons per acre. As it is impossible to foresee where small areas of high density may develop, it may be good judgment to provide for a higher density of population in designing the lateral sewers than in designing the sub-mains, and a higher density in sub-main design than in that of the main or trunk sewers.

The volume of industrial wastes depends upon the character and magnitude of the industries. For entire cities it has been estimated to vary from a negligible amount to as high as 80 gal. per capita per day, and for small areas it may amount to several hundred gallons per capita per day. In determining the allowance to be made in sewer design a forecast is necessary of the class of industries which may be expected, and studies and measurements of the wastes from similar existing industries should be made. In a district of varied industries an allowance of 10 000 gal. per acre per day for industrial wastes, will probably prove sufficient. Where tanneries, stockyards, packing houses, laundries, canning factories and similar industries are to be expected, special study should be made.

Mercantile or commercial districts yield more sewage per unit of area than residential or industrial districts, primarily because of the concentration during the business portion of the day, of many persons in office buildings, stores, etc. Gagings of the flow from such districts have shown rates in excess of 100 000 gal. per acre per day.

While it is often the aim of the sewer builder to construct sewers which will not admit large amounts of ground water, it is almost impossible to prevent some intrusion of such water, especially through connections, and allowance for this must be made in computing the capacity of separate sewers. The amount of ground water entering a sewer will depend upon the length and possibly the size of the sewer, the number of connections, the character of the surface and soil formations, and upon the quality of workmanship secured in construction. It may vary from a negligible amount to as high as 40 000 gal. or more per day per mile of sewer. There have been measurements which indicated that the infiltration of ground water exceeded 100 000 gal. per day per mile of sewer. In general for sewers of modern construction 50 000 gal. per day per mile of sewer may be considered an ample allowance, although conditions may indicate that a larger or smaller allowance is desirable for portions of a system.

Design of Separate Sewers. Before the design can be developed the basic data must be determined along the lines indicated above.

In the design of a system of separate sewers a topographic map is desirable of a scale of 100 or 200 ft. to the in., showing contours, brooks, rivers, ponds and streets. The contours should be sufficiently close to allow the designer to plot profiles of streets with reasonable accuracy, i. e., where the surface slope is 6 per cent. or less the map should show contours at 2 ft. intervals; where the surface slope is much greater than this, 5 ft. intervals will usually suffice. Summits in streets should be marked and the elevations given to tenths of a foot, as should also points of depression or " pockets."

Having determined the basic data to be used in the design, much labor can be saved by constructing curves showing the maximum quantity of sewage for a given area. It is also advantageous and advisable to arrange the computations in tabular form, as indicated on page 51, "Sewerage and Sewage Disposal "* Where practicable the minimum velocity when flowing full or half full, should be 2.5 ft. or more per second. If the velocity is much lower than this, deposits may form, requiring periodic cleaning of the sewers.

For computing the capacity of sewers the Kutter formula is that most widely used. The minimum size should be not less than 6 in. and preferably 8 in. in diameter. In order to reach a rational solution the following steps are required —

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*A Text Book by Leonard Metcalf and Harrison P. Eddy. McGraw-Hill Book Co., New York, 1922.

1. Draw lines on the topographic map to represent the sewer in each street.
2. Locate the manholes.
3. Sketch the limits of drainage areas, for each lateral sewer.
4. Measure the drainage areas.
5. Plot a profile of the line which is to serve the lowest point or the controlling line as to elevation.
6. Prepare a table of computations.

Table 1 gives the computation for the design of the main sewer for a residential district which is to receive in addition the wastes from two small industrial districts.

Design of Combined Sewers or Drains. In the combined system the rate of discharge of domestic sewage and industrial wastes is so small compared to the rate of run-off of storm water (usually being less than the error in estimating storm water) that the former is neglected and the sizes are determined from the estimate of storm flow alone.

When precipitation occurs some of the water evaporates, some of it percolates into the ground becoming ground water, another portion is held back by vegetation, some is required to fill depressions in the surface, and the remainder runs off. It is the last portion for which provision must be made in combined sewers or drains. The portion of the whole precipitation which runs off during a storm depends upon —

1. The character of the surface — whether pervious or impervious, and whether or not it is frozen.

By "impervious" is meant surfaces covered with roofs or pavements.

"Pervious" surfaces are those covered with lawns or gardens.

2. The roughness of the surface.

With smooth surfaces there are fewer depressions to be filled before run-off begins.

3. The steepness of the surface slope.

On steep slopes the run-off concentrates into streams quicker than on flat slopes and is therefore retarded less also, in general, depressions and surface irregularities have less capacity for storage on steep slopes than on flat slopes.

4. The character of the so-called pervious portions, i.e. whether of a clayey or sandy nature.

5. The rate or intensity of precipitation.

The last factor — intensity of precipitation — is the most important, yet many of the early engineers appear to have given little if any consideration to it. In fact in several American cities formulas were used for the design of combined sewers, which had been used for European cities where the run-off allow-

TABLE I - COMPUTATIONS FOR SEPARATE SEWER

NUMBER OF LINE	MANHOLE NUMBER		LOCATION. Street	LENGTH Feet	AREA. Acres		SEWAGE AND GROUND WATER from curve$^\phi$ M.G.D	INDUSTRIAL WASTES AT 20,000 gal per acre daily	TOTAL MAX. RATE. SEWAGE GROUND WATER ETC.		SIZE OF SEWER Inches dia.	CAPACITY C.F.S	VELOCITY F.P.S.	SLOPE Feet per foot	SURFACE ELEVATION Upper end	SURFACE ELEVATION Lower end	INVERT ELEVATION	
	From	To			INCREMENT	TOTAL			M.G.D.	C.F.S.							Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	6	7	Eloise St	400	16.0	16.0	0.6	-	0.6	0.9	8	0.9	2.5	.009	182.8	182.4	173.75	173.39
2	7	8	Sumner Ave	425	24.5	40.5	1.3	-	1.3	2.0	12	2.0	2.5	.005		181.9	173.06	172.85
3	8	9	" "	395	14.5	55.0	1.6	-	1.6	2.5	15	3.1	2.5	.0035		182.0	172.60	172.46
4	9	10	" "	415	19.7	74.7	2.0	-	2.0	3.1	15	3.1	2.5	.0035		181.6	172.46	172.31
5	10	11	" "	390	27.3	102.0	2.6	-	2.6	4.0	15	4.0	3.3	.0058		182.6	172.31	172.08
6	11	12	" "	480	34.0	136.0	3.0	-	3.0	4.7	15	4.7	3.8	.008		181.7	172.08	171.70
7	12	13	White St.	365	127.0*	263.0	5.1	2.5	7.6	11.7	24	11.7	3.7	.0037		181.5	170.95	170.81
8	13	14	" "	390	39.2	302.2	5.6	-	8.1	12.5	24	12.5	4.0	.0043		180.8	170.81	170.64
9	14	15	" "	410	44.8	347.0	6.4	-	8.9	13.8	27	13.8	3.5	.0028		180.9	170.39	170.27
10	15	16	" "	425	27.0	374.0	7.0	-	9.5	14.7	27	14.7	3.7	.0031		180.4	170.27	170.14
11	16	17	Portland St	500	187.0*	561.0	9.3	3.8	15.6	24.1	30	24.5	5.0	.0035		180.7	169.89	169.72
12	17	18	" "	405	29.3	590.3	9.6	-	15.9	24.6	30	24.5	5.0	.0035		180.1	169.72	169.58
13	18	19	" "	370	47.7	638.0	10.0	-	16.3	25.2	30	25.2	5.1	.0037		180.4	169.58	169.44
14	19	20	Oakland St.	365	294.0	932.0	13.3	-	19.6	30.3	33	30.0	5.0	.0031		181.1	169.19	169.08
15	20	21	" "	395	16.9	948.9	13.6	-	19.9	30.8	33	31.0	5.1	.0033		180.9	169.08	168.92
16	21	22	" "	400	41.1	990.0	14.2	-	20.5	31.7	33	32.0	5.3	.0035		179.8	168.92	168.78

* Industrial Areas. ϕ Flow curve not reproduced herewith.

ance was $\frac{1}{2}$ in. per hour. Sewers designed and constructed on this basis were frequently subjected to overloading and it gradually became recognized that in the United States provision for higher rates must be made. Subsequently many attempts were made to devise a formula for computing the rate of run-off. These were usually expressed in the general form of

$$Q = A\,C\,i \sqrt[x]{\frac{S}{A}},$$

where

A is the area expressed in acres.
C is a coefficient usually expressed as a decimal.
i is the intensity or rate of rainfall, in in. per hour.
S is the surface slope in ft. per 1 000 ft. and
Q is the run-off in cu. ft. per second.

Some of these formulas have been widely used — for instances the Burkli-Ziegler, which is $Q = A\,C\,i \sqrt[4]{\frac{S}{A}}$, and the McMath, which is $Q = A\,C\,i \sqrt[5]{\frac{S}{A}}$. When these formulas were derived little data were available concerning the intensity of rainfall. Beginning about the year 1897, however, the U. S. Weather Bureau has established automatic recording rain gages in the principal cities of the country, and in recent years the number has been greatly increased. One of the longest automatic records is that of the Central Park gage, in New York City which covers a period of over fifty years. The rain gage at Chestnut Hill has been in use for about forty-five years. There are now available considerable data for single stations, and engineers have become more and more impressed with the importance of rainfall intensity and its relation to the rate of run-off.

During recent years the empirical formulas have been less used and have been to a large extent supplanted by the so-called Rational Method. This method is expressed by the general formula $Q = CiA$, where

Q	the run-off in cu. ft. per second.
C	the coefficient of run-off.
A	the drainage area in acres, and
i	the intensity of rainfall in in. per hour.

Analysis of rainfall records shown that the intensity of precipitation is a function of the duration of the downpour.

For Boston the formula $i=\frac{20.4}{t^{0.61}}$* represents intensities to be expected on the average once in 15 years, and this is called the "fifteen year frequency curve." In this formula i is the intensity in in. per hour and t is the duration in minutes. The New York fifteen-year curve is $i=\frac{185}{t+17}$† and gives values from 20 to 50 per cent. higher than the Boston curve.

Most of the intense rains occur during storms of the so-called thunderstorm type, in which the highest rates usually come at or near the beginning of the storm, and this fact is recognized and allowed for in the Rational Method. The use of this method requires the determination of the following basic data —

1. The rainfall frequency curve for which provision should be made.
2. The probable future condition of the drainage area, i.e. the percentage of impervious surface which may be expected when the district shall have been developed to the extent assumed.
3. The run-off coefficient, i.e., the proportion of the rainfall which will run off over the surface of the ground.
4. The probable time required for water to flow over the surface of the ground to the first inlet, called the "inlet time" or "time of entrance."
5. The area tributary to the sewer at the point at which the size is to be determined.
6. The time required for water to flow in the sewer from the first inlet to the point in question, which added to the inlet time, or time of entrance, gives the time of concentration.
7. The application of the proper run-off coefficient to the rainfall curve for the time of concentration, from which the rate of run-off per unit of area may be computed.

In determining the rainfall frequency curve for which provision should be made consideration must be given to the financial condition of the city and to the amount of inconvenience and damage which would result from flooding. Most rainfall records covering a period of twenty years or more contain one or two storms of such severe intensity that they might be classed by the courts as "Acts of God." The expense of providing for such storms is prohibitive in some communities. Hence it may be considered wise to provide only for storms likely to be expected on the average once in 5, 10, or 15 years. In districts which do not contain real estate of high value or valuable merchandise subject to damage, and where little inconvenience would be experienced from occasional flooding, it will probably be wise to provide for only a 5-year or perhaps even for only a 2-year frequency rainfall.

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* See Journal Boston Society Civil Engineers Vol. 7, 1920 p. 47 Maximum Rates of Precipitation at Boston for Various Frequencies of Occurence, by Harrison P. Eddy.

† See Trans. Soc. Mun. Eng. of N. Y., 1913.

The probable future condition of the drainage area may be predicted from its location and topography (i. e. whether suited to residential, industrial or commercial uses) and from the character of existing developments. Where zoning laws are in effect the character will be controlled to some extent and the future condition can be fairly closely foreseen. Residential districts of lots say 50 ft. by 100 ft., with single houses, may have from 40 to 50 per cent. of impervious surface. With larger lots the percentage will be lower. Commercial and industrial areas may develop to as high as 80 or 90 per cent. of impervious surface, although in the case of railroad yards the percentage is usually very low. Parks and cemeteries may be considered as having from 10 to 15 per cent. imperviousness.

From the percentage of impervious surface the run-off coefficient may be estimated. A recent modification in the method of determining the run-off coefficient is the so-called "zone principle of run-off," the basic assumptions of which are as follows:

1. That the most intense rates of rainfall occur at the beginning of a storm and are of short duration.
2. That the surface is substantially dry at the beginning of the rain.
3. That as the rainfall continues the proportion of the precipitation which runs off gradually increases.
4. That the maximum run-off occurs when the entire tributary area is contributing water to the point in question.

5. That the rate of rainfall is uniform over the entire drainage area.

6. That the rate of rainfall is uniform from the beginning of the storm and for a duration equal to the time of concentration.

The fifth assumption is believed by some to be too stringent, and excellent data in support of this belief were presented by Mr. Frank A. Marston in an address before the Sanitary Section of the Boston Society of Civil Engineers, October 5, 1921. The data as yet, however, are not sufficient to warrant a decision as to the proper coefficient which should be applied, and it is hoped that such studies will be carried further and more data collected with the aid of a greater number of rain gages than are now installed, and with closer coöperation.

The first two assumptions — i. e. that the most intense rates occur at the beginning of the storm, and that the surface is substantially dry at the beginning, while not true for all storms are practically so for the majority of storms of great intensity.

The assumption that the rate of run-off increases with the duration of the downpour is obviously true, for the following reasons —

Nearly all surfaces absorb some moisture, and before run-off can begin they must become wet. This means that some water is actually stored on all surfaces — perhaps an amount varying from 0.01 to 0.02 of an in. or more in depth. Practically all surfaces also contain depressions in which the first surplus water unabsorbed collects. Some depth of water likewise is required in gutters, in the joints between units of pavements, and in the sewers themselves, before any appreciable velocity of flow is realized. Evaporation is constantly taking place even during the progress of a rain, and on pervious or semi-pervious surfaces water is percolating into the subsoil. As the rain continues the depressions become filled and a less proportion is thus retained in storage; also, as the surfaces become saturated less water is absorbed. This is true not only of pervious but also of impervious surfaces, but the higher the degree of perviousness the longer the time required for saturation, and some soils such as coarse sand overlying other coarse soil, will absorb water about as fast as it falls, even in very intense storms. The amount of water required to fill gutters and sewers, while actually running off is in effect in temporary storage. Data regarding the rate or increase of run-off, is rather meager. Some measurements have been made in St. Louis, under the direction of Mr. W. W. Horner, Chief Engineer of the Division of Sewers and Paving, who has suggested the following coefficients:

For Pervious Surfaces.

For rains of 5 min. duration	0.10
10 ,, ,,	0.20
15 ,, ,,	0.30
30 ,, ,,	0.40
60 ,, ,,	0.50
90 ,, ,,	0.55

For Impervious Surfaces.

For rains of 5 min. duration	0.50
10 ,, ,,	0.60
15 ,, ,,	0.70
20 ,, ,,	0.80
30 ,, ,,	0.85
45 ,, ,,	0.90
60 ,, ,,	0.95

Consider now a hypothetical drainage area (Fig. 1) which has the shape of a sector of a circle, with the point of concentration at the center of the circle. Assume that the direction of flow is radial and that the velocity is uniform over the entire district. Consider the district as divided into zones of equal width. Let V = the velocity in some unit per minute, and α the angle of the sector. Assume that each zone is of such width that it requires 5 min. for water to flow across the zone. Then the areas of the zones starting with the one at the center, will have the following ratios to each other — 25 : 75 : 125 : 175 : 225 : 275 : 325, etc. The area shown is assumed to have a time of concentration of 45 min. Consider now the flow at the center, for different intervals of time after the beginning of the rain. At the end of 5 min. the run-off from the first zone *only* will have reached the point of concentration, but the run-off from each of the other zones will have reached the outer edge of the zone next nearest the point of concentration. At the end of 10 min. water which fell on the first and second zones will be flowing at the center, but that portion coming from the first zone will be at a higher rate per unit of area than that from the second zone, as at the end of 10 min. the water which has reached the center from the first zone fell *after* the first 5 min. of the rain, whereas that coming from the second zone fell *during* the first 5 min. Consider that the storm lasts 45 min., then the flow at the center will be made up of run-off from rain which fell during different portions of the storm. That portion coming from the first zone is the run-off from rain which fell after 40 min., that from the second zone from rain which fell after 35 min., that from the third zone from rain which fell after 30 min., and so on, so that that portion coming from the farthest zone is the run-off coming from rain which fell during the first 5 min. of the storm. By combining the proportional area of each zone with its proper run-off coefficient we are able to determine the mean coefficient for the entire area. The computations and results for pervious surfaces are shown in Table 2 and for impervious surfaces are shown in Table 3. The mean coefficient for a

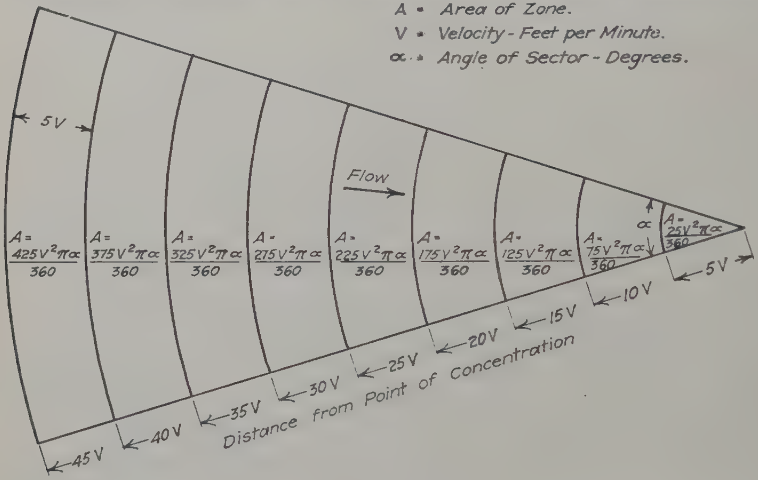

A = Area of Zone.
V = Velocity - Feet per Minute.
α = Angle of Sector - Degrees.
5 V
Flow
A = $\frac{425V^2\pi\alpha}{360}$
A = $\frac{375V^2\pi\alpha}{360}$
A = $\frac{325V^2\pi\alpha}{360}$
A = $\frac{275V^2\pi\alpha}{360}$
A = $\frac{225V^2\pi\alpha}{360}$
A = $\frac{175V^2\pi\alpha}{360}$
A = $\frac{125V^2\pi\alpha}{360}$
A = $\frac{75V^2\pi\alpha}{360}$
A = $\frac{25V^2\pi\alpha}{360}$
α
5 V
10 V
15 V
20 V
25 V
30 V
35 V
40 V
45 V
Distance from Point of Concentration

FIG. 1. — HYPOTHETICAL DRAINAGE DISTRICT ILLUSTRATING THE ZONE PRINCIPLE OF RUN-OFF.

DURATION MINUTES "t"	RUN-OFF COEFFICIENT F ONE ZONE AFTER RAI FALL OF DURATION STATE "C"	AREA-RATIO OF ZONES "A"	CUMULATIVE ARE RATIO OF ZONES "ΣA"="t2"	Duration or Period of Concentration - Minutes																							
				180		150		135		120		105		90		75		60		45		30		20		10	
				C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A
5	.10	25	25	.70	17.5	.65	16.2	.625	15.6	.60	15.0	.575	14.4	.55	13.7	.525	13.1	.50	12.5	.45	11.3	.40	10.0	.35	8.8	.20	5.0
10	.20	75	100	.692	51.9	.642	48.1	.617	46.3	.592	44.4	.567	42.6	.542	40.7	.517	38.8	.483	36.2	.433	32.5	.375	28.2	.30	22.5	.10	7.5
15	.30	125	225	.683	85.5	.633	79.1	.608	76.0	.583	73.9	.558	69.8	.533	66.6	.508	63.5	.467	58.5	.417	52.1	.35	43.8	.20	25.0		
20	.35	175	400	.675	118.1	.625	109.4	.60	105.0	.575	100.7	.55	96.3	.525	91.9	.50	87.5	.45	78.8	.40	70.0	.30	52.5	.10	17.5		
25	.375	225	625	.667	150.0	.617	138.8	.592	133.1	.567	127.5	.542	122.0	.517	116.2	.483	108.5	.433	97.5	.375	84.4	.20	45.0				
30	.40	275	900	.658	181.0	.608	167.2	.583	160.4	.558	153.5	.533	146.6	.508	139.7	.467	128.5	.417	114.6	.35	96.2	.10	27.5				
35	.417	325	1225	.65	211.2	.60	195.0	.575	187.0	.55	178.8	.525	170.7	.50	162.5	.45	146.2	.40	130.0	.30	97.5						
40	.435	375	1600	.642	240.8	.592	222.0	.567	212.5	.542	203.0	.517	194.0	.483	181.0	.433	162.3	.375	140.5	.20	75.0						
45	.45	425	2025	.633	269.0	.583	248.0	.558	237.3	.533	226.4	.508	216.0	.467	198.5	.417	177.2	.35	148.8	.10	42.5						
50	.467	475	2500	.625	297.0	.575	273.0	.55	261.2	.525	249.5	.50	237.5	.45	214.0	.40	190.0	.30	142.5								
55	.483	525	3025	.617	324.0	.567	298.0	.542	284.1	.517	271.8	.483	254.0	.433	227.5	.375	197.0	.20	105.0								
60	.50	575	3600	.608	350.0	.558	321.0	.533	306.5	.508	292.2	.467	268.4	.417	240.0	.35	201.2	.10	57.5								
65	.508	625	4225	.60	375.0	.55	344.0	.525	328.0	.50	312.5	.45	281.1	.40	250.0	.30	187.5										
70	.517	675	4900	.592	399.6	.542	366.0	.517	349.0	.483	320.0	.433	292.5	.375	253.0	.20	135.0										
75	.525	725	5625	.583	423.0	.533	386.5	.508	368.2	.467	338.5	.417	302.5	.35	254.0	.10	72.5										
80	.533	775	6400	.575	445.5	.525	407.0	.50	387.5	.450	349.0	.40	310.0	.30	232.5												
85	.542	825	7225	.567	467.5	.517	426.5	.483	398.5	.433	357.5	.375	309.3	.20	165.0												
90	.55	875	8100	.558	488.5	.508	445.0	.467	409.0	.417	365.0	.35	306.0	.10	87.5												
95	.558	925	9025	.55	508.5	.50	462.5	.45	416.3	.40	370.0	.30	277.5														
100	.567	975	10000	.542	528.5	.483	471.0	.433	422.5	.375	365.5	.20	195.0														
105	.575	1025	11025	.533	546.6	.467	479.0	.417	427.5	.35	359.0	.10	102.5														
110	.583	1075	12100	.525	565.0	.45	484.0	.40	430.0	.30	322.5																
115	.592	1125	13225	.517	582.0	.433	487.5	.375	422.0	.20	225.0																
120	.60	1175	14400	.508	597.0	.417	490.0	.35	411.2	.10	117.5																
125	.608	1225	15625	.50	612.5	.40	490.0	.30	367.5																		
130	.617	1275	16900	.483	616.0	.375	475.0	.20	255.0																		
135	.625	1325	18225	.467	619.0	.35	464.0	.10	132.5																		
140	.633	1375	19600	.45	619.0	.30	412.5																				
145	.642	1425	21025	.433	617.0	.20	285.0																				
150	.65	1475	22500	.417	615.0	.10	147.5																				
155	.658	1525	24025	.40	610.0																						
160	.667	1575	25600	.375	591.0																						
165	.675	1625	27225	.35	568.8																						
170	.683	1675	28900	.30	502.5																						
175	.692	1725	30625	.20	345.0																						
180	.70	1775	32400	.10	177.5																						
Σ (C×A)				14716		9642		7550		5744		4208		2934		1909		1122		561		207		73.8		12.5	
RUN-OFF COEFFICIENT $\frac{\Sigma(C\times A)}{\Sigma A}$				.454		.429		.414		.399		.382		.362		.339		.312		.277		.230		.185		.125	

TABLE 3 - RUN-OFF COEFFICIENTS FOR IMPERVIOUS SURFACES ... SECTOR SHAPED AREAS.

DURATION MINUTES "t"	RUN-OFF COEFFICIENT FOR ONE ZONE AFTER RAIN-FALL OF DURATION STATED "C"	AREA - RATIO OF ZONES "A"	CUMULATIVE AREA - RATIO OF ZONES "ΣA"="t²"	Duration or Period of Concentration - Minutes																							
				180		150		135		120		105		90		75		60		45		30		20		10	
				C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A	C	C×A
5	.50	25	25	.95	24	.95	24	.95	24	.95	24	.95	24	.95	24	.95	24	.95	24	.90	23	.85	21	.80	20	.60	15
10	.60	75	100	"	71	"	71	"	71	"	71	"	71	"	71	"	71	.933	70	.883	66	.825	62	.70	53	.50	38
15	.70	125	225	"	119	"	119	"	119	"	119	"	119	"	119	"	119	.917	115	.867	109	.80	100	.60	75		
20	.80	175	400	"	167	"	167	"	167	"	167	"	167	"	167	"	167	.90	158	.85	149	.70	123	.50	88		
25	.825	225	625	"	214	"	214	"	214	"	214	"	214	"	214	.933	210	.883	199	.825	186	.60	135				
30	.85	275	900	"	261	"	261	"	261	"	261	"	261	"	261	.917	252	.867	239	.80	220	.50	138				
35	.867	325	1225	"	309	"	309	"	309	"	309	"	309	.95	309	.90	293	.85	276	.70	228						
40	.883	375	1600	"	356	"	356	"	356	"	356	"	356	.933	350	.883	331	.825	309	.60	225						
45	90	425	2025	"	404	"	404	"	404	"	404	"	404	.917	390	.867	369	.80	340	.50	213						
50	.917	475	2500	"	451	"	451	"	451	"	451	.95	451	.90	428	.85	404	.70	333								
55	.933	525	3025	"	498	"	498	"	496	"	498	.933	490	.883	464	.825	434	.60	315								
60	.95	575	3600	"	547	"	547	"	547	"	547	.917	527	.867	498	.80	460	.50	288								
65	"	625	4225	"	594	"	594	"	594	.95	594	.90	563	.85	531	.70	438										
70	"	675	4900	"	641	"	641	"	641	.933	630	.883	597	.825	558	.60	405										
75	"	725	5625	"	689	"	689	"	689	.917	667	.867	629	.80	581	.50	362										
80	"	775	6400	"	737	"	737	.95	737	.90	698	.85	659	.70	543												
85	"	825	7225	"	784	"	784	.933	770	.883	727	.825	682	.60	496												
90	"	875	8100	"	831	"	831	.917	802	.867	759	.80	700	.50	438												
95	"	925	9025	"	879	.95	879	.90	833	.85	787	.70	648														
100	"	975	10000	"	927	.933	911	.883	862	.825	806	.60	587														
105	"	1025	11025	"	974	.917	942	.867	887	.80	820	.50	513														
110	"	1075	12100	"	1021	.90	969	.85	914	.70	751																
115	"	1125	13225	"	1069	.883	995	.825	928	.60	675																
120	"	1175	14400	"	1118	.867	1019	.80	941	.50	588																
125	"	1225	15625	.95	1163	.85	1041	.70	857																		
130	"	1275	16900	.933	1191	.825	1057	.60	765																		
135	"	1325	18225	.917	1215	.80	1061	.50	663																		
140	"	1375	19600	.90	1239	.70	964																				
145	"	1425	21025	.883	1260	.60	856																				
150	"	1475	22500	.867	1281	.50	738																				
155	"	1525	24025	.85	1299																						
160	"	1575	25600	.825	1299																						
165	"	1625	27225	.80	1300																						
170	"	1675	28900	.70	1173																						
175	"	1725	30625	.60	1035																						
180	"	1775	32400	.50	888																						
		Σ(C×A)			28029		19124		15301		11923		8971		6443		4340		2666		1419		579		236		53

TABLE 4.

Run-off Coefficients, Zone Principle.

Per Cent. of Impervious Surface.		* 00		10		20		30		40		50		60		70		80		90		100	
		†		0		10		20		30		40		50		60		70		80		90	
T‡	"i"§	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci	c	ci
10	6.91	0.125	0.86	0.165	1.14	0.205	1.42	0.245	1.69	0.285	1.97	0.325	2.24	0.365	2.52	0.405	2.80	0.445	3.08	0.485	3.36	0.525	3.63
20	4.30	0.185	0.80	0.225	0.97	0.266	1.14	0.306	1.32	0.346	1.49	0.387	1.66	0.427	1.84	0.467	2.01	0.507	2.18	0.548	2.36	0.588	2.53
30	3.25	0.230	0.75	0.271	0.88	0.312	1.01	0.354	1.15	0.395	1.28	0.436	1.42	0.477	1.55	0.518	1.68	0.560	1.82	0.601	1.95	0.642	2.09
45	2.47	0.277	0.69	0.319	0.79	0.362	0.89	0.404	1.00	0.446	1.10	0.489	1.21	0.531	1.31	0.573	1.42	0.615	1.52	0.658	1.63	0.700	1.73
60	2.03	0.312	0.63	0.355	0.72	0.398	0.81	0.440	0.89	0.483	0.98	0.526	1.07	0.569	1.16	0.612	1.24	0.654	1.33	0.697	1.42	0.740	1.50
75	1.75	0.339	0.59	0.382	0.67	0.425	0.74	0.469	0.82	0.512	0.89	0.555	0.97	0.598	1.04	0.641	1.12	0.685	1.20	0.728	1.27	0.771	1.35
90	1.54	0.362	0.56	0.405	0.62	0.449	0.69	0.492	0.76	0.535	0.83	0.579	0.89	0.622	0.96	0.665	1.03	0.708	1.09	0.752	1.16	0.795	1.23
105	1.39	0.382	0.53	0.425	0.59	0.468	0.65	0.511	0.71	0.554	0.77	0.598	0.83	0.641	0.89	0.684	0.95	0.727	1.01	0.770	1.07	0.813	1.13
120	1.26	0.399	0.50	0.442	0.56	0.485	0.61	0.528	0.67	0.571	0.72	0.614	0.77	0.656	0.83	0.699	0.88	0.742	0.94	0.785	0.99	0.828	1.05
135	1.17	0.414	0.48	0.457	0.53	0.499	0.58	0.542	0.63	0.584	0.68	0.627	0.73	0.670	0.78	0.712	0.83	0.755	0.88	0.797	0.93	0.840	0.98
150	1.09	0.429	0.47	0.471	0.51	0.513	0.56	0.555	0.60	0.597	0.65	0.640	0.70	0.682	0.74	0.724	0.79	0.766	0.84	0.808	0.88	0.850	0.93
180	0.96	0.454	0.44	0.495	0.48	0.536	0.52	0.577	0.55	0.618	0.59	0.660	0.63	0.701	0.67	0.742	0.71	0.783	0.75	0.824	0.79	0.865	0.83

* For sector shaped areas.
† For rectangular shaped areas.
‡ Period of concentration — min.
§ Intensity of precipitation — in. per hr.

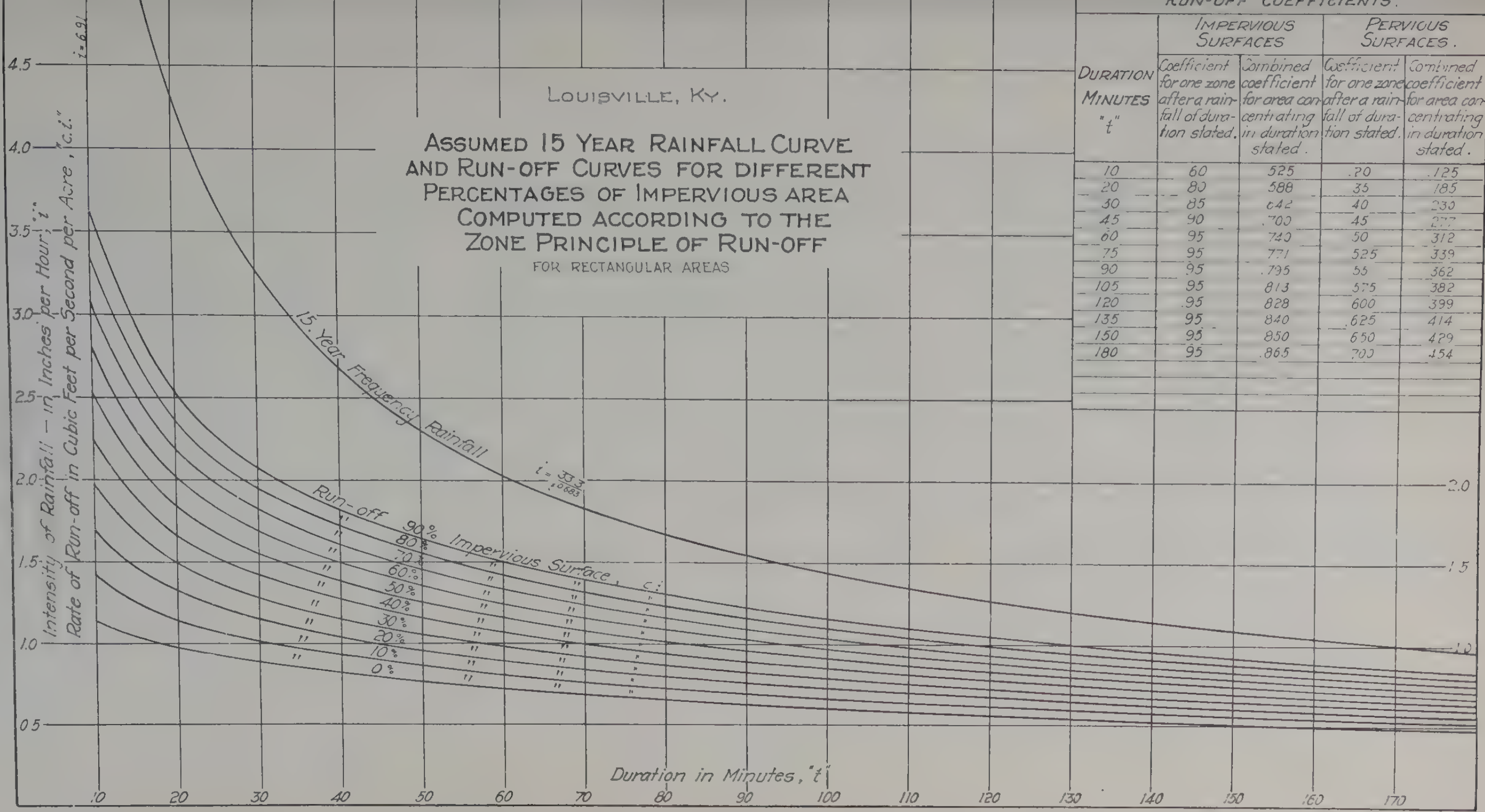
sector for a given period of concentration, is shown in the bottom line of the tables. By combining the proper coefficients with the different proportions of pervious and impervious surfaces the coefficient for any degree of required imperviousness may be found. These are shown in Table 4. As in practice it seldom happens that areas are sector-shaped, but are more nearly rectangular, and as rectangular shaped areas have slightly higher run-off coefficients than sector-shaped areas, the coefficients as computed have been increased 10 per cent. On Fig. 2 are shown the 15-year rainfall curve for Louisville, Kentucky, and run-off curves — or Ci curves — for different percentages of impervious surfaces for rectangular-shaped areas. From these curves the run-off in inches per hour, or approximately in cubic feet per second per acre can be read directly for any given time of concentration. These are the Values of Ci which are to be used in the formula $Q = CiA$.

The inlet time, or time of entrance, depends upon the surface slope and the percentage of impervious surface, and is commonly taken at from 10 to 20 min., the shorter time being used for commercial districts.

The time required for water to flow in the sewers is simply a matter of computation and is found by dividing the length by the velocity. The time of concentration is the inlet time plus the time of flow in the sewers.

Application of Rational Method. For designing a system of combined sewers or drains a map similar to that required for separate sewer design is needed. The procedure is to lay out the system as in separate sewer design, locating manholes and sketching the limits of drainage areas, and measuring them. With this problem the district must be subdivided much smaller than in the previous problem, as a small pipe will accomodate a much smaller area. In general, the drainage area must be determined at each manhole.

Start at the upper end of the longest line and plot a profile of the ground surface, following down through the lateral, submains and trunks to the point of discharge. Scales of 100 ft. to the in. horizontal, and 10 ft. vertical, will be sufficiently close for preliminary designs.. For the first stretch look up the Ci



RUN-OFF COEFFICIENTS.

DURATION MINUTES "t"	IMPERVIOUS SURFACES		PERVIOUS SURFACES.	
	Coefficient for one zone after a rainfall of duration stated.	Combined coefficient for area concentrating in duration stated.	Coefficient for one zone after a rainfall of duration stated.	Combined coefficient for area concentrating in duration stated.
10	60	525	.20	.125
20	80	588	35	185
30	85	642	40	230
45	90	.700	45	277
60	95	740	50	312
75	95	771	525	339
90	95	.795	55	362
105	95	813	575	382
120	.95	828	600	399
135	95	840	.625	414
150	95	850	650	429
180	95	.865	700	454

FIG. 2.

PLACE, LOUISVILLE, KENTUCKY.

TABLE 5.

COMPUTATIONS FOR DRAINS OR COMBINED SEWERS

DISTRICT, 34th St. Woodland to Garland.

RAINFALL CURVE, $i = \frac{135}{t+10}$ 15 year frequency.

DATE, 6-13-21 SHEET No. 1 OF 1 SHEET

MAIN DRAIN

SUB-MAIN

COMPUTED BY, F.S.B. CHECKED BY J.W.R.

CAPACITY OF DRAINS HOW DETERMINED, Kutter, $n = .015$ 24" & under, $n = .013$ over 24"

No	Line		Name of Street	Area Increment		Cumulative Area		C i.	Q = C i A. CFPS	Time-Minutes		Velocity FPS.	Capacity CFPS	Diameter (Inches)	Slope of Drain in Feet		Surface Elev.		Invert Elev.		Aver Cut in feet	Length feet	REMARKS
	From Manhole	To Manhole		Acres	Coef of Imp	Acres	Av Coef of Imp			Elapsed Upper End	Flow in Section				Rate per ft	Total	Upper End	Lower End	Upper End	Lower End			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)
1	1	2	Garland Av.	1.7	.30	1.7	.30	1.72	2.8	15.0	1.4	2.7	3.3	15	.004	.88	449.5	448.6	442.19	441.31		220	
2	2	3	" "	3.0	"	4.7	"	1.67	7.8	16.4	1.3	3.6	7.8	20	.0045	1.26		448.3	440.89	439.63		280	
3	3	4	" "	3.0	"	7.7	"	1.63	12.5	17.7	1.4	3.1	12.5	27	.00165	.45		448.0	439.05	438.60		270	
4	4	5	" "	1.6	"	9.3	"	1.58	14.6	19.1	0.7	3.7	14.6	27	.0022	.35		447.5	438.60	438.25		160	
5	5	6	36th St.	6.4	"	15.7	"	1.57	27.7	19.8	2.3	3.5	24.7	36	.00135	.66		447.6	437.50	436.84		490	
6	6	7	" "	20.1	"	35.8	"	1.51	54.1	22.1	1.6	5.0	55.	45	.002	.96		446.9	436.09	435.13		480	
7	7	8	" "	17.3	"	53.1	"	1.48	78.7	23.7	1.8	5.0	78.	54	.00155	.84		446.2	434.38	433.54		540	
8	8	9	Grand Ave.	7.1	"	60.2	"	1.43	86.1	25.5	1.6	5.0	88.	57	.00145	.71		444.3	433.29	432.58		490	
9	9	10	" "	4.4	"	64.6	"	1.40	90.5	27.1	1.5	5.1	91.	57	.00155	.73		444.6	432.58	431.85		470	
10	10	11	34th. St.	15.1	"	79.7	"	1.36	108.4	28.6	1.7	5.0	110.	63	.0013	.65		444.0	431.55	430.70		500	
11	11	12	" "	10.6	"	90.3	"	1.33	120.0	30.3	1.6	5.1	122.	66	.00125	.61		443.3	430.45	429.84		490	
12	12	13	" "	12.0	"	102.3	"	1.29	132.0	31.9	1.4	5.8	135.	66	.00155	.76		443.0	429.84	429.08		490	
13	13	14	" "	20.3	"	122.6	"	1.27	155.7	33.3	1.2	6.6	156	66	.00205	.96		442.6	429.08	428.12		470	
14	14	15	" "	19.0	"	141.6	"	1.25	177.0	34.5		7.5	177.	66	.0027	1.30		440.0	428.12	426.82		480	

curves and read the run-off for the duration corresponding to the time of entrance and the percentage of impervious surface. The value found, multiplied by the area, will give the rate of run-off in cu. ft. per second, for which the sewer must provide. Next approximate the slope of the sewer by inspection of the profile; then find the proper size, from diagrams of the capacity of conduits, noting the velocity. Next divide the length to the next manhole below, by the velocity, and this will give the time of flow in the sewer, which should be added to the inlet time to give the time of concentration for designing the next stretch of sewer. Proceed on down the line, determining the required size of each stretch separately.

It is practically essential that computations of design by the Rational Method be made in tabular form similar to that shown in Table 5.

The controlling conditions in problems of sewer design will be different for each locality. The wide variations among the data of run-off and sewage flow indicate the need for special study for individual cases and the proper application of such data in determining the basic assumption for design must be predicated upon experience and judgment always taking into consideration all of the local conditions.

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FOUNDATIONS FOR THE NEW FIRST NATIONAL BANK OF BOSTON.

By Howard K. Alden.*

(Presented before the Designers' Section, October 11, 1922.)

The new bank and office building which Stone & Webster are constructing for The First National Bank of Boston is being designed by York & Sawyer, Architects, of New York City, who have retained Mr. H. G. Balcom, also of New York City, as their structural engineer.

The building is shaped like a blunt wedge, 107 ft. to 160 ft. wide, 260 ft. long and 125 ft. high. There will be ten stories above the street and two below. The bank will occupy the two basements, the ground floor, and four floors directly above. The five upper stories will be rented for office use.

The lowest basement floor varies in depth from 33 ft. to 49 ft. below the curb. This is the deepest basement of any size that has been constructed in Boston, and the foundations which are of necessity deeper still, present many interesting problems.

During 1920, before wrecking operations were started, soil borings were made under the Equitable Building to determine the nature of the material that would be encountered in the foundation work. These borings (Fig. 1) were carried down through the clay by means of a soil augur with an extension pipe handle. The augur was operated inside of a 2-in. steel pipe driven into the ground as the boring progressed. No water whatever was used in making these borings, and no difficulty was experienced in bringing up samples of the soil until the overburden of clay had been penetrated and the point of the augur entered sand. From this point down to a hard stratum of boulder clay samples were obtained by driving a length of 1-in. pipe ahead of the casing and pulling this up when it was full of new material.

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* Construction Engineer, Stone and Webster, Boston, Mass.

The borings taken showed uniform material overlying the boulder clay which occurred at approximately grade — 35 or about 55 ft. below the sidewalk surrounding the property.

The boulder clay encountered at this grade resisted further driving of the boring pipe almost as effectively as ledge rock, and was within such a reasonable distance of the bank's lower basement that it seemed wise to carry the foundations for so important a building to this stratum.

All borings showed a layer of fine sand from 5 ft. to 11 ft. in thickness a short distance above the boulder clay, but there were no indications in the borings that any excessive amount of water existed in this sand and for this reason no difficulty was anticipated in establishing concrete foundations on hard material.

Underpinning Massachusetts Trust Building.

On account of some uncertainty in the dates for terminating the tenants' leases in the old John Hancock Building, it was deemed advisable to start underpinning the party wall between the Massachusetts Trust Building and the new bank before the wrecking of the old John Hancock Building could be commenced, so that a good portion of this underpinning work might be finished ahead of the general excavation and foundations for the main building.

Two methods were considered for this work; one for driving pre-cast piles under the party wall and capping them with a continuous concrete girder directly underneath the existing grillage. The other proposal contemplated constructing independent piers, or caissons, forming practically a solid concrete wall resting on boulder clay from Federal Street to Devonshire Street.

Col. Charles R. Gow was present at several conferences in which the merits of the two methods were discussed, and furnished many helpful suggestions.

The contract for this underpinning work was awarded to the Chas. R. Gow Co., and the preliminary work of excavating for this underpinning was started during May.

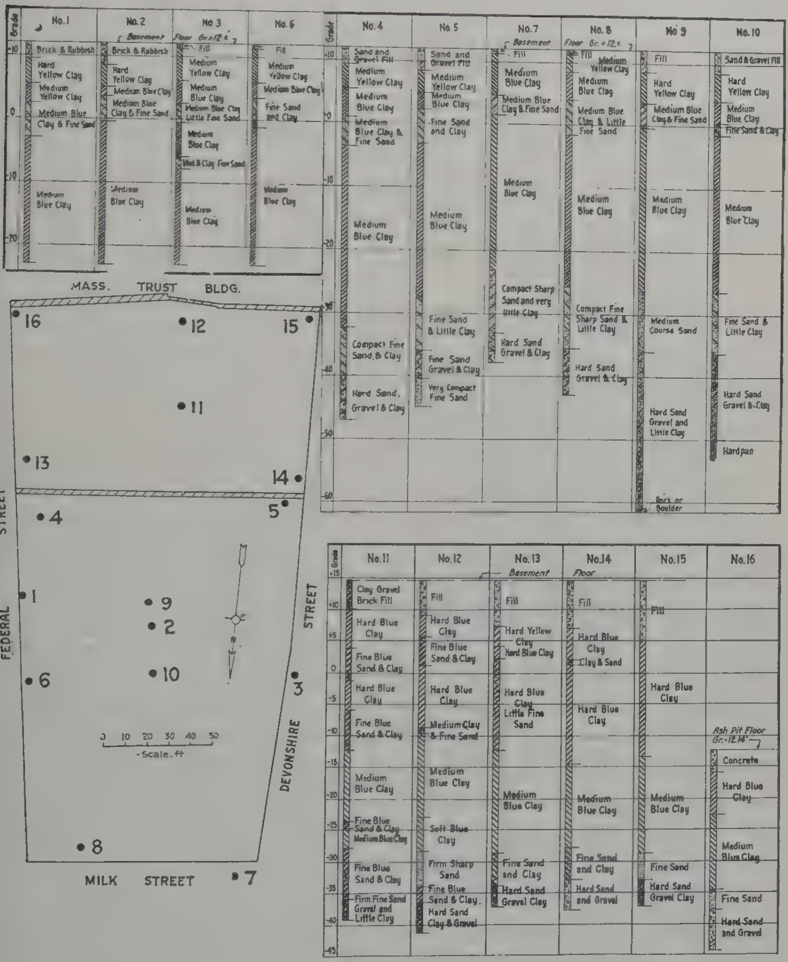

FIG. 1. — TEST BORINGS — LOCATION DIAGRAM AND SECTIONS.

The working trench was made 10 ft. wide and opened up for nearly the entire length of the party wall. It was started in the basement of the old John Hancock Building and carried to the top of the existing grillage at grade —14.

While excavating this trench the projecting granite blocks of the original wall footings at grade +5 had to be split off from the John Hancock side. A similar projection was removed 10 or 12 years ago on the Massachusetts Trust side of the wall when it was previously underpinned during the construction of the Massachusetts Trust Building. An 8-in. course of brick, protecting a layer of tar and felt waterproofing, was removed from the bank side of the party wall below the granite block foundation.

Below these granite blocks there were brick and masonry walls resting on a grillage approximately 2 ft. thick and 10 ft. to 15 ft. wide of concrete in which 12 in. and 15 in. structural "I" beams were bedded.

The subcontract included the removal of all projecting portions of the existing grillage and the construction of underpinning piers or caissons from that grillage into boulder clay at grade — 37. (Fig. 2.)

The first caisson 7 ft. long and 6 ft. wide was started by excavating a sheeted pit 7 ft. long, 9 ft. wide and 5 ft. deep, so located that a section 7 ft. long and 4½ ft. wide came under the wall with the balance in the open. The section of chamber under the wall was carefully sheeted so that no slippage of earth could take place. In this working chamber a concrete shell 7 ft. long, 6 ft. wide, 4 ft. high and 1 ft. thick was constructed with a 4 in. x 4 in. x ⅜ in. angle iron cutting edge around the outside perimeter. After this shell had set up sufficiently, the inner forms were removed and mucking was started inside. Four 15-ton jacks were installed on the top edge of the shell and arranged to operate against the concrete grillage above in order to force the shell down as rapidly as the earth was removed from the inside and from under the cutting edge.

After this hollow caisson had been jacked down 4 ft., excavation was discontinued and 4 ft. more of shell was built in place. This process was repeated three or four times until the water bearing stratum of sand was reached where pumping was required at such a rate it was considered hazardous to the Trust Building foundations.

The caisson was then allowed to fill with water to approximately grade − 18, and a diver employed to continue the excavation through the remaining 8 ft. or 10 ft. of sand to boulder clay. This method was necessarily slow and expensive, but unquestionably safe.

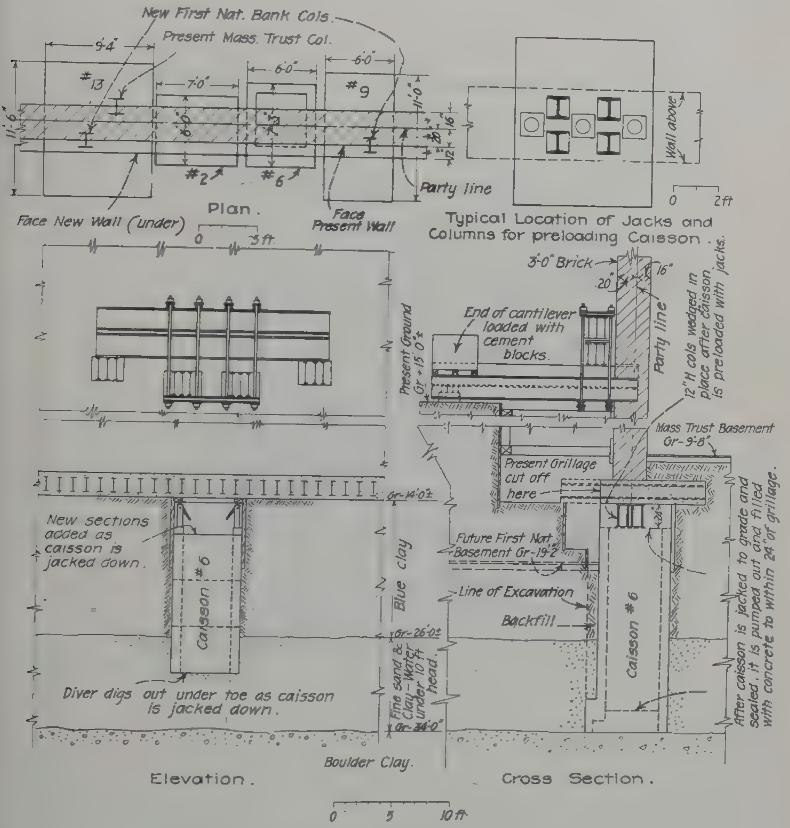
New First Nat. Bank Cols.
Present Mass. Trust Col.
9'-4"
7'-0"
6'-0"
6'-0"
11'-6"
11'-0"
16"
20"
12"
#13
#2
#6
#9
Party line
Face New Wall (under)
Face Present Wall
Plan.
0 5 ft.
Wall above
0 2ft
Typical Location of Jacks and Columns for preloading Caisson.
3'-0" Brick
20"
16"
Party line
End of cantilever loaded with cement blocks.
12" H cols wedged in place after caisson is preloaded with jacks.
Present Ground Gr + 15' 0"±
Mass Trust Basement Gr-9'-8"
Present Grillage cut off here
24"
Gr-14'0"±
New sections added as caisson is jacked down.
Caisson #6
Future First Nat. Basement Gr-19'-2"
Line of Excavation
Backfill
Blue clay
Gr-26'0"±
Fine sand & Clay - Water under 10 ft head.
Gr-34'-0"
Caisson #6
After caisson is jacked to grade and sealed, it is pumped out and filled with concrete to within 24" of grillage.
Diver digs out under toe as caisson is jacked down.
Boulder Clay.
Elevation.
Cross Section.
0 5 10 ft

FIG. 2. — TYPICAL UNDERPINNING CAISSON.

When hard boulder clay was reached the diver removed all soft or loose material, levelled the area within the cutting edge, wherever this was possible without removing large boulders, and left a reasonably smooth surface for concreting.

The caisson was then filled by lowering bags of concrete into the water which the diver emptied and distributed inside the shell, filling a section about 4 ft. in height. This acted as a seal permitting the balance of the caisson to be pumped out and filled by chuting on the following day, to a point approximately 2 ft. below the under side of the grillage. After allowing the concrete in the caisson to set 3 or 4 days, hydraulic jacks were applied to produce an initial load 25 per cent. in excess of the designed load. An attempt was made to maintain this overload by means of jacks 12 to 16 hours, but it proved unsuccessful on account of a slight though constant leakage to which all hydraulic jacks invariably seem susceptible.

Instead of using jacks for an indefinite period, it was decided to hold the jack load only until it could be transferred, by slim wedges, to structural steel struts with top and bottom bearing plates.

When these struts got their load, there was no possibility of leaking or slippage. As a precaution, however, the jack load was again applied after a few hours time and the wedges once more driven home. The results obtained by this method on caisson No. 6, about 6 ft. wide and 7 ft. long, having an area of 43.5 sq. ft., are shown below:

Caisson No. 6. Bearing Area. 43.5 sq. ft.

Time.	Load.	Total Settlement.	Rebound.
10.00 A.M.	326 tons by 3 hydraulic jacks	.045 ft.	
10.05 A.M.	Entire load transferred to four 12-in. H cols. 24 in. long by steel wedges and jacks released......................	.037 ft.	.008 ft.
1.15 P.M.	326 tons by jacks	.048 ft.	
1.30 P.M.	Wedges redriven and jacks removed....	.042 ft.	.006 ft.

Design Load = 261 tons.
Jack Load Imposed = 326 tons, or 25 per cent. overload.

Prior to starting any one caisson, cantilever shores were installed each side to relieve the area being undermined of a portion of its load, thus reducing the danger of settlement during construction.

To date seven of these caissons, or nearly 40 per cent. of the total work, have been completed. Check levels taken at frequent intervals along the north side of the party wall show no settlement for half its length and a maximum settlement of 3/16 in. in the other half.

FOUNDATIONS.

The foundations for the new building were first designed as typical Gow piles with bases belled out at an angle of 60° with the vertical. This type of construction seemed best adapted to the conditions; but after encountering so much water in the underpinning work, a test caisson was driven near the center of the bank before definitely deciding on the method to be adopted. This caisson was carried down by the open method to grade −25 where water-bearing sand was encountered which showed a decided tendency to boil and required excessive pumping.

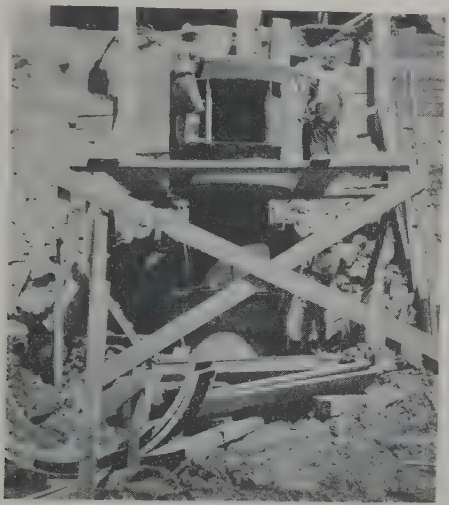

FIG. 3. — CONCRETE CAISSON NO. 45 SHOWING AIR LOCK IN PLACE.

When this stage was reached, the work was temporarily discontinued and an air lock installed. The caisson was then carried down without further difficulty to boulder clay at grade −35, using air pressures from 10 to 12 lb. in excess of normal atmospheric pressure.

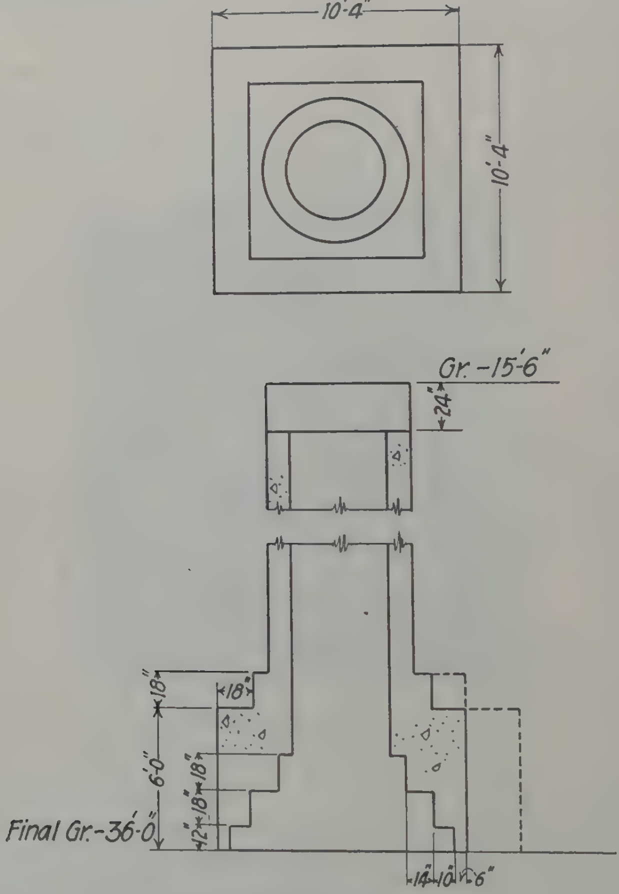

FIG. 4 — TYPICAL INTERIOR CAISSON.

This test proved conclusively that the open cylinder method generally used in this vicinity for sinking Gow piles was impracticable on this particular job.

It was then decided to install all the remaining caissons in a manner similar to that employed for the test caissons. (Fig. 3 and Fig. 4).

Four other methods of constructing foundations for the bank were considered but careful analysis showed them to be either more expensive or more hazardous than the air caissons selected. These methods were namely:

(1) Wood piles capped with concrete at grade −15 to −30, depending on the grade of the basements in the various sections of the bank. This method showed a slight saving but was abandoned for the reason that complete saturation of the soil into which the piles would be bedded was considered doubtful on account of the varying levels in the basement and the danger of draining the upper levels while pumping the sub-soil to relieve pressure against the waterproofing.

Furthermore, the risk involved in driving piles for the exterior footings adjacent to paved streets and near high pressure fire lines, together with the added hazard of handling pile-driving equipment on the bracing and the difficulty of spotting piles properly through sections of heavy shoring were determining factors in the decision to abandon wood piles.

(2) Concrete piles capped with concrete at similar grades were considered, but abandoned because of even greater expense and probable danger to banks from excessive jarring.

(3) Concrete piers constructed on boulder clay in open pits surrounded by steel sheet piling driven into the hard stratum. Abandoned on account of higher cost and the slow progress necessitated by working in so restricted an area.

(4) Spread footing or mat underneath the entire building just below the column bases. Abandoned because the basement grades were such that part of this mat would rest on clay and part on the water bearing stratum of fine sand. These two different materials probably would have produced unequal settlement throughout the building.

The caisson work progressed rapidly and twelve foundations were in various stages of completion with four or five down to grade when unexpected soil conditions required tests that somewhat delayed work.

From information which could be obtained, no soil tests had ever been made on this boulder clay as it is called by the Building Department. While it was generally assumed that such material was good for a unit load of 7 to 8 tons per sq. ft. no reliable data could be found. It was, therefore, considered important to establish a satisfactory bearing value for this soil.

The first soil test was made on an unconfined area 12 in. sq. at grade −36.5 under one of the large vault caissons with air pressure removed and relatively little water over the area being tested. The test showed a settlement of 3 in. in 24 hours under a load of 5 tons per sq. ft., 4 in. in 48 hours under a $7\frac{1}{2}$ ton load, and 5 in. in 84 hours with the same load maintained. This result was highly unsatisfactory and very disturbing to all parties concerned.

Soil Test No. 1.

Area tested — 1 sq. ft. of soil at grade −36.5 under Footings 36 and 36B.
No attempt made to confine soil around area tested.

	Time.	Load.		Total Settlement in Feet.	
Aug. 5, 1922.	3.00 P.M.	2.4 tons	(Platform)	0.000	Initial Reading.
	3.20 "	5.4 tons	(Platform and Pig Iron)	0.030	
	4.00 "	5.4 tons	"	0.046	
	5.00 "	5.4 tons	"	0.060	
	6.05 "	5.4 tons	"	0.084	
	6.30 "	5.4 tons	"	0.096	
Aug. 6, 1922	9.10 A.M.	5.4 tons	"	0.249	
	12.10 P.M.	5.4 tons	"	0.250	
	5.50 "	5.4 tons	"	0.253	
Aug. 7, 1922	7.45 A.M.	5.4 tons	"	0.254	
	9.00 "	5.4 tons	"	0.254	
	9.30 "	7.6 tons	"	0.317	
	11.00 "	7.6 tons	"	0.330	
	1.00 P.M.	7.6 tons	"	0.335	
	4.00 "	7.6 tons	"	0.339	
	5.00 "	7.6 tons	"	0.343	Water pumped from pit to examine bottom. Immediate settlement of $\frac{3}{8}$ in.
	6.30 "	7.6 tons	"	0.384	
	10.00 "	7.6 tons	"	0.401	
Aug. 8, 1922	9.15 A.M.	7.6 tons	"	0.410	
	5.00 P.M.	7.6 tons	"	0.412	
Aug. 9, 1922	9.00 A.M.	7.6 tons	"	0.419	Test Discontinued.

A second test was made on an area 2 ft. sq. at grade −36.5 in the bottom of one of the other large vault caissons. Around the area being tested there was a space approximately 6 in. wide of unconfined soil. The balance of the caisson was sealed with a 2-ft. mat of concrete. The 2-ft. square area was subjected to a test load of 5 tons per sq. ft. Settlement of 2½ in. occurred before 75 per cent. of the total load had been applied. Settlement continued gradually to a maximum of 3 in. when the test apparently came to rest. The result of this test was no more encouraging than the previous test had been.

An inspection of the material in the 6-in. space outside the area being tested showed a soft yielding mass into which a man's foot would sink several inches and a ¾-in. rod would penetrate two or three feet, of its own weight. The material bore no resemblance to the hard unyielding boulder clay in which the caisson excavation had stopped while the air pressure held the ground water back. Under those conditions the boulder clay flaked off under a sharp miner's pick with difficulty, and a ¾-in. rod driven with a man's full strength made very little impression on the surface.

Soil Test No. 2.

Area tested — 4 sq. ft. of soil at grade —36.5 under Footings 36 and 36B. 6-inch space around area tested not confined.

	Time.	Load.	Total Settlement in Feet.
August 8, 1922.........	12.45 A.M.	1.68 tons (Platform)	.044
	1.45 "	17.34 tons	.218
	2.00 "	17.34 tons	.220
	2.30 "	17.34 tons	.220
	2.45 "	17.34 tons	.220
	3.00 "	17.34 tons	.220
	3.30 "	17.34 tons	.220
	5.30 "	17.34 tons	.2205
	6.30 "	17.34 tons	.222
	7.30 "	17.34 tons	.223
	8.30 "	17.34 tons	.223
	9.30 "	17.34 tons	.224
	10.30 "	17.34 tons	.225
	11.30 "	17.34 tons	.225
	1.00 P.M.	17.34 tons	.225

Load increased to 20 tons. During loading of additional iron, the platform was wedged to prevent vibration.

Time.	Load.	Total Settlement in Feet.
2.00 P.M.	20 tons total	.229
2.30 "	20 tons total	.231
3.00 "	20 tons total	.231
4.00 "	20 tons total	.234
5.00 "	20 tons total	.234
8.00 A.M.	20 tons total	.236
9.00 "	20 tons total	.236

Arrangements were made to run a third test under conditions approximating the natural load conditions to which the soil under the building foundations would be subjected.

The third test was applied on a plate 17½ in. square, or 2 sq. ft. in area, carefully bedded on boulder clay in the bottom of the caisson under column 24. A concrete sealing mat 3 ft. in thickness was placed in this caisson with a rectangular opening 16 in. square centered on the steel bearing plate, into which the post carrying the loading platform could later be set. The test plate and concrete seal were both installed under air pressure. As soon as the concrete had hardened, the air lock was removed, the loading platform set in position, and the caisson completely filled with water to restore in every way possible normal soil conditions at the bottom of the test plate.

A loading of 20 tons equivalent to 10 tons per sq. ft. was applied gradually to this steel plate and resulted in a maximum settlement of less than 1 in.

SOIL TEST NO. 3.

Area tested — 2 sq. ft. of soil at grade —36 under caisson No. 24.
All soil around area tested carefully confined and normal load conditions were restored.

	Time.	Load.	Total Settlement. in Feet.	
August 9, 1922...	1.45 P.M.	1.2 tons (platform)	0.000	Initial Reading.
	2.00 "	2.2 tons	0.001	
	2.10 "	4.2 tons	0.013	
	2.20 "	6.2 tons	0.021	
	2.30 "	8.2 tons	0.028	
	2.40 "	10.0 tons	0.036	
	3.30 "	10.0 tons	0.039	
	4.00 "	10.0 tons	0.039	
	5.00 "	10.0 tons	0.039	
	5.20 "	11.0 tons	0.041	
	6.00 "	13.0 tons	0.047	
	6.20 "	15.0 tons	0.054	
	7.00 "	15.0 tons	0.056	
	8.00 "	15.0 tons	0.057	
August 10, 1922...	8.30 A.M.	15.0 tons	0.057	
	2.15 P.M.	15.0 tons	0.057	
	2.40 "	16.0 tons	0.058	
	3.25 "	18.0 tons	0.060	
	4.00 "	20.0 tons	0.064	
	4.15 "	20.0 tons	0.068	
	5.15 "	20.0 tons	0.069	
August 11, 1922...	8.30 A.M.	20.0 tons	0.079	
	1.00 P.M.	20.0 tons	0.079	
	2.00 "	20.0 tons	0.079	Test Discontinued.

This test was felt to be very satisfactory and it seemed fair to assume that the settlement to be expected under the large caissons would be materially less than that developed under this small test plate.

While these tests proved that the soil was capable of supporting 7 to 8 tons per sq. ft. without excessive settlement, the importance of the banking structure and its intimate relation to the Massachusetts Trust Building seemed to warrant adopting a more conservative soil loading in order to reduce settlement to a minimum. Consequently, a unit loading of $5\frac{1}{2}$ tons per sq. ft. for the interior columns and 5 tons per sq. ft. for the exterior was established.

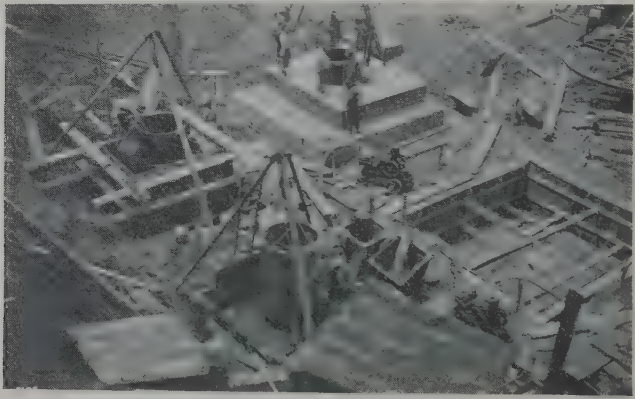

FIG. 5. — INTERIOR CAISSONS IN VARIOUS STAGES OF CONSTRUCTION.

After reaching this decision, the caissons already excavated were filled and the remaining caissons in the interior of the Milk Street end were opened up. (Fig. 5.)

The caisson work is started by constructing the bell ends in pits excavated approximately 6 ft. into the clay below the basement floor. The bells are concreted and a section of the shaft constructed above the bell before the excavation is carried further. When the bell concrete has set up sufficiently, the inside forms are stripped and muckers commence removing earth from under the bell and its cutting edge. A portion of this excavated material is carried away, but a large percentage is deposited on the bell to increase its weight.

As the excavation proceeds, the caisson sinks of its own weight. It is kept plumb by wedging against the sides of the original pit and by undercutting more rapidly at the points which show a tendency to lag.

The excavation is carried down in the open until the water bearing stratum of sand is reached. Here the work is stopped temporarily until air locks can be installed in the shaft and air connections made to the compressed air lines.

The excavation is then continued in a similar manner except that the muckers work under air pressure of 10 to 18 lb. and elevate their material in heavy canvas bags instead of buckets.

The caissons under air are lowered by releasing the air pressure momentarily and permitting the huge mass to slip down on account of its tremendous weight. When boulder clay is reached, the caisson is plumbed, the bottom carefully levelled off, inspected by the resident engineer and a sealing mat of concrete approximately 2 ft. thick deposited over the caisson bottom by passing the concrete into the air lock and then discharging the contents of the air lock into the caisson.

The sealing mat is allowed to set up 24 hours under air pressure, at a temperature of approximatly 80° Fahr. The air lock is then removed and the balance of the caisson concreted.

To date practically all the interior caissons for the north end of the bank have been finished and work on several wall caissons is in progress.

It requires about ten working days to complete one caisson, but several caissons are, of course, operated simultaneously.

Air for the caisson work is furnished by three motor driven, heavy duty Ingersoll Rand air compressors, so connected that one machine is a spare or emergency machine for replacing either of the other two when repairs must be made, or for supplementing the other two in case of unusual demand.

With average weather conditions, it is expected that all foundations and a large portion of the basement walls for the bank will be finished before the end of the present year.

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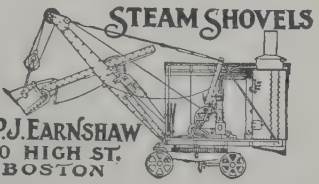
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